CHAPTER 17

SUBSTRUCTURES

The design compressive strengths of concrete are:

- Reinforcing steel shall conform to AASHTO M31M, Grade 60.
- The minimum cover for reinforcing steel is 2 in. for formed concrete.
- Where concrete is placed against soil, the minimum cover is 3 in.

17.1 Architectural Treatments

Architectural treatments are used to improve aesthetics of bridges in the District. Any of these treatments or an approved treatment will be required. Treatments include:

- Brick facing
- Stone facing
- Exposed aggregate
- Form-liners

Stone facing will be required on historic structures or when directed by the SHPO and Commission on Fine Arts. Form-liners are available simulating various textures and treatments. They have been used to simulate stone and brick and can be considered on a case-by-case basis. Form-liners provide architectural treatment at lower cost than other types of treatments. Brick or simulate brick facing are generally used on property walls on neighborhood streets where new wall is necessary due to street construction.

17.2 Substructure Protection

DDOT routinely requires piers and abutment areas located under expansion joints as well as exposed ends of concrete piers and abutments to be coated. Use water-based epoxy or silane or approved method to protect substructure concrete. Generally, vandalism in the form of graffiti on bridge substructures will be reviewed to determine its potential as a target for graffiti vandalism, and, if it is needed, an anti-graffiti coating will be specified.

17.3 Substructure Drainage

Any water that accumulates behind abutment back walls and retaining walls must be drained to prevent settlement of the embankment or failure of the wall. This is accomplished through footing drains, weep holes, and geosynthetic drains. Granular backfill behind the walls is essential to carry the water to footing drains and weep holes.

Footing drains are preferred instead of weep holes to drain walls that are visible to the public. A perforated drainpipe is installed behind the footing with outlets located to minimize aesthetic impacts. Weep holes may be used in walls that are not generally visible to the public, such as in back walls for stream crossings. Under no circumstances shall the water be drained onto the sidewalks and roadways.

17.4 Abutment Design

A well-designed abutment provides safety against the possibility of overturning about the toe of the footing, against sliding on the footing base and against crushing of foundation material or overloading of piles. Service loads are used in checking the stability of the substructure. Upon satisfying all stability requirements, the design process analyzes the strength of the abutment components. This analysis is made utilizing the Load Factor Design (LFD) method.

Abutments support the end spans of the bridge and retain the approach roadway embankment. The types of abutments used in are:

- Cantilever
- Stub And
- Integral

The designer must evaluate the foundation conditions below the bottom of the footing. Where foundation conditions are acceptable, abutments on spread footings are permitted. Under conditions where the foundation soil cannot support the loads, piles are used to support the footing.

The designer must consider possible alternative construction sequences to assure that all loads applied to piles are considered in the design. Specifically, the placement of fill after the piles are driven can cause down-drag on the piles. It is possible to develop negative skin friction in some soils, and the designer must consider auguring through these soils to preclude this condition. Battered piles should be avoided in negative skin friction situations because of the additional bending forces imposed on the piles. Down-drag can be reduced by specifying coating of that portion of the pile subject to down-drag.

17.5 Cantilever Abutments

Cantilever abutments are commonly used in the District. The wall provides for the reactions from the superstructure and also resists the thrust from the earth backfill. It is designed to resist this thrust as a retaining wall, cantilevered from the footing.

17.6 Stub Abutments

Stub abutments are used in situations where the need for retaining is minimal. Stub abutments are rarely used in the District and they are built on pile foundations. In these instances, the bridge seat acts as a pile cap and must be sufficient to carry, by beam action, the loads from the bridge superstructure to the pile foundation. A proprietary retaining wall may be placed in front of the abutment.

17.7 Design Loads

The forces acting on an abutment are summarized below. This example is for a cantilevered abutment. The forces acting on a stub abutment are similar.

17.7.1 Forces Acting on Abutments

- Substructure Dead Load - Weight of concrete = 150 Lb/ft3
  - Weight of Earth resting on substructure. Use 120 Lb/ft^3 for compacted earth behind the abutment. The fill in front of the abutment is usually disregarded.
  - Active Earth Pressure. Refer to the AASHTO **Standard Specifications for Highway Bridges** or use minimum an equivalent fluid pressure (mass) of 40 $Lb/ft^2/ft$.
  - Superstructure Dead Load. Include future wearing surface (10 Lb/ft^2). Distribute superstructure reaction over length of the abutment.
  - Live Load on Approaches. Use a 2 ft. Live Load Surcharge on all surfaces subject to highway traffic (refer to the AASHTO **Standard Specifications for Highway Bridges**).

  NOTE: Where an approach slab is used, the Live Load Surcharge is not considered. The weight of the approach slab is omitted from the calculations.

- Superstructure Live Load
  - Number of Design Traffic Lanes (refer to the AASHTO **Standard Specifications for Highway Bridges**). Use 2 lanes on 20 to 30 ft. pavement. Use 3 lanes on 30 to 40 ft. pavement. Use 4 lanes on 40 to 50 ft. pavement, etc.
  - Reduction in Load Intensity (refer to the AASHTO **Standard Specifications for Highway Bridges**); for one or two lanes, use 100 percent; for three lanes use 90 percent; for four lanes or more, use 75 percent.
  - Longitudinal Force. Use 5 percent of the live load in all lanes carrying traffic headed in the same direction (refer to the AASHTO

Standard Specifications for Highway Bridges). Distribute over the length of the abutment between the faces of wingwalls, along the bearing line and acting in the same direction.
- Impact - Refer to AASHTO **Standard Specifications for Highway Bridges**.
- Wind - Refer to AASHTO **Standard Specifications for Highway Bridges.**
- Thermal Forces - Refer to AASHTO **Standard Specifications for Highway Bridges.**
- Uplift - Refer to AASHTO **Standard Specifications for Highway Bridges**.
- Buoyancy - Refer to AASHTO **Standard Specifications for Highway Bridges**.
- Earth Pressure - Refer to AASHTO **Standard Specifications for Highway Bridges**, except that an equivalent fluid pressure of 40 $Lb/ft^2/ft$ shall be the minimum.
- Earthquake - Refer to AASHTO **Standard Specifications for Highway Bridges**.

There are various combinations of these loads and forces that act on the abutment. The abutments shall be designed to safely withstand all applicable combinations. Refer to AASHTO **Standard Specifications for Highway Bridges**.

17.8 Retaining Wall Design

Retaining walls are designed to withstand lateral earth and water pressures including live and dead load surcharges, the weight of the wall, temperature and shrinkage effects, and earthquake loads in accordance with AASHTO **Standard Specifications for Highway Bridges.**

17.8.1 Retaining Wall Types

There are four basic types of retaining wall structures available for the designer to consider in a specific design. These are:

- Reinforced concrete walls are constructed using cast-in-place or precast concrete elements. They may be constructed on spread footings or founded on piles. They derive their capacity through combinations of dead weight and structural resistance.
- Proprietary retaining walls are patented systems for retaining soil. Most common systems used are: gravity and mechanically stabilized.
- Gravity walls generally use interlocking, soil-filled reinforced concrete bins to resist earth and water pressures. They depend on dead weight for their capacity. Mechanically stabilized walls use metallic or

polymeric tensile reinforcement in the soil mass and modular precast concrete panels to retain the soil. Gravity walls are generally constructed for property walls on the neighborhood streets where the construction of footings or any portion of the wall is constrained due to ROW.

- Tied back walls consist of piles driven at designated spacings and then tied back using drilled or grouted type anchors. The spaces between the piles are spanned with structural elements, such as wood, reinforced concrete lagging, precast or cast-in-place concrete panels or steel members, to retain the soil.

17.8.2 Safety Factors and Design Criteria

All walls will be designed using the following design criteria and safety factors:

- Factor of safety - Overturning: 2.0
- Factor of safety - Sliding: 1.5
- Weight of fill: 120 Lb/ft3
- Equivalent hydrostatic pressure: 40 lb/ft^2/ft
- Passive pressure resistance to sliding from soil in front of the wall will not be considered.

17.8.3 Proprietary Retaining Walls

Consideration of economics, location, construction requirements and aesthetics should be included in the evaluation. All abutments constructed behind proprietary retaining walls will be founded on piles. Spread footings will not be permitted. Proprietary retaining walls are used to retain earth and do not carry vertical structure loads.

Each design location must be evaluated based on the specific merits (advantages and disadvantages) of the specific construction being considered. Close consideration must be made for long-term stability, stream flow and storm flows. Positive erosion control in addition to geo-technical fabric is needed. Do not use these walls in locations where water might reach the wall.

When proprietary retaining walls are included in a project, special provisions must be included in the contract documents to guide the suppliers. The wall suppliers provide all required engineered designs of the structural wall. Suppliers' designs are included in the plans. The contractor selects a supplier's design and submits a bid accordingly.

Proprietary wall designs shall be in accordance with the current **AASHTO Standard Specifications for Highway Bridges**. As per Sections 5.2.1.4

and 5.2.1.5 of the **AASHTO Standard Specifications for Highway Bridges**, the following limitations should be adhered to in the proposed use of alternate proprietary Mechanically Stabilized Earth Walls (MSE) and Prefabricated Modular Walls:

- MSE walls should not be used under the following conditions:
  - When utilities other than highway drainage must be constructed within the reinforced zone.
  - With galvanized metallic reinforcements exposed to surface or ground water contaminated by acid mine drainage or other industrial pollutants as indicated by low pH and high chlorides and sulfates.
  - When floodplain erosion may undermine the reinforced fill zone, or where the depth of scour cannot be reliably determined.
  - Prefabricated modular systems shall not be used under the following conditions:
    -- On curves with a radius of less than 800 ft., unless the curve can be substituted by a series of chords.
    -- When calculated longitudinal differential settlements along the face of the wall are greater than 1/200.

NOTE: Steel modular systems shall not be used where the ground water or surface runoff is acid contaminated or where de-icing spray is anticipated.

17.8.4 Proprietary Wall Design

For projects in which proprietary retaining wall structures are deemed feasible, the Designer will analyze site conditions during preliminary engineering and make recommendations regarding which wall system may be used.

Conceptual wall plans hereafter referred to as control plans, shall be provided in the final Contract Plans and shall include project specific details. Complete detailed proprietary wall drawings will not be included in the contract documents. After the award of the contract, complete proprietary wall plans for the selected wall will be prepared by the proprietor and submitted by the Contractor as shop drawings in accordance with the DDOT standards. A set of original drawings will be added to the record set of the contract documents after approval of the shop drawings.

If site conditions warrant that only one proprietary manufacturer can be used, the Designer shall request and obtain approval to prepare complete plans for the suitable wall type. For such an occurrence, sole source justification is required. A waiver, as per the requirements of **23.CFR 635.411**, must be obtained from the FHWA. Special site conditions shall include, but not be limited to, the following (see next page):

- Excessive height of wall (more than 30 ft.)
- Poor foundation conditions (low allowable bearing pressure)
- Constructability
- Noise barriers mounted to wall
- Longitudinal drainage in the common structure volume
- Obstructions such as sign structures

The Control Plans shall include, but not be limited to, the following information:

- Plan and elevation views of the wall(s): The Elevation view of wall(s) shall show existing and proposed ground lines, elevations at 25 ft. intervals at the top of wall and proposed ground line (used to compute quantities), wall embedment (maximum elevation at top of leveling pad) and beginning and end of wall stations.
- Control data for horizontal and vertical alignment
- Specific/nominal limits of the wall(s)
- Locations of existing and proposed utilities
- Boring locations
- General Notes:
  - ROW limits / construction easements
  - If warranted, construction sequence requirements, traffic control, access, and stage construction sequence
  -Work Item Quantities table
  - Estimate of Quantities Table
  - Limits of Common Structure Volume
  - Limits and requirements for drainage features within the Common Structure Volume, limits and requirements that will affect the construction or stability of the wall beneath, on top of, and behind the retaining wall.
  - At stream location, high water and normal water levels and scour protection
  - Design parameters (safety factors), which shall include, but not be limited to, the following:
    -- Allowable Bearing Capacity
    -- Soil Unit Weight
    -- Angle of Internal Friction
    -- Anticipated settlement
    -- If required, Foundation Subgrade Treatment
    -- Magnitude, location and direction of external loads due to bridges, sign structures, traffic surcharge, etc.
    -- Seismic criteria
    -- Sections through wall showing offset control point, pay area, ditches, sidewalks, superelevation and any unusual features:

-- General details showing:

--- End of wall interfaces

--- Wall/coping/barrier or barrier interfaces

--- Drainage pipe and inlet details, slip joint details

--- Compatibility with roadway plans

--- Excavation, , cofferdam requirements

--- Architectural details (such as dimensional requirements, special wall features; such as facing finish, texture, color or planting)

--- Location and size of any existing or proposed structures

--- Location of overhead signs or roadway lighting

--- Location and height of noise barrier, if applicable

-- Foundation Report and Recommendation:

--- When alternate retaining walls are to be included in a project, the Foundation Report shall provide complete detailed information as to the reason for recommendation of alternate type retaining wall systems. The Designer shall evaluate global external stability, sliding, overturning, slope stability, bearing pressure, settlement.

--- The Report shall indicate the maximum elevation at the top of leveling pads or footings and the design foundation pressures at those elevations.

--- If soil subgrade treatment, soil enhancement, and/or unsuitable material removal is required, the Report shall clarify such recommendations along with potential effects that the recommendations may have on the various alternates.

--- In order to permit the availability of the Report to the Contractor, the Designer shall assure that the most current Report is provided to the Project Manager.

- When the allowance of alternate type proprietary walls is permitted, the contractor shall be responsible:
  - Providing the design calculations and construction plans for the proprietary wall systems. The calculations shall include internal stability verification of the wall system.
  - In accordance with the Standard Specifications, drawings and design calculations shall be submitted for review. Once the submission is found to be acceptable, the Contractor shall submit final signed and sealed design calculations, one (1) set of mylars and the required number of signed and sealed prints as per the Standard Specifications.

- The Designer will sign and seal these mylars noting that the walls are checked for external stability and for conformance with the design concept of the project. Also, he will modify the Index of Drawings on the Contract set of plans.
- An additional set shall be furnished if Railroad structures are involved.
- A note on the Control Plan shall be provided specifying which type of proprietary wall is to be constructed at each wall location.

The Contractor shall submit detailed shop and working drawings including the design calculations. Complete information as to the proposed method of fabrication and erection of precast units and related components shall be provided. Shop drawings shall be prepared and submitted in accordance with the Standard Specifications. The Department reserves the right to reject any alternate wall system or details which do not conform to the control plans, pre-approved details, **Standard Specifications or AASHTO Specifications.**

17.9 Steel Sheet Piles Wall

The contractor is responsible for the design of temporary structures, with approval of the designs by the Department. Sheet piling walls may be either cantilever or anchored design. In anchored design, deadmen are constructed, and the sheeting wall is anchored to them using tie rods. In no situation will an abutment be constructed using driven sheet piling as support for the structural loads. A690 sheet piles should be used in marine environments. A709 Grade 250 and A709 Grade 345 sheet piles are used in non-marine environments. Both types are always coated. In cases where steel sheeting is used with laggings as permanent construction, a coating is required. Where a cap is required, a concrete cap is preferred. Designers should refer to the **AISC Sheet Piling Design Manual**.

Steel sheeting below the top of the seal concrete will generally be left in place. If sheeting is left in place it shall be anchored to the top of the seal concrete.

17.10 Piers

17.10.1 Waterways

When a pier is located in a marine environment, reinforcement steel (including footing bars and dowels) shall be zinc-coated (hot-dipped galvanized) or epoxy coated. The Designer shall designate the use of either galvanized or epoxy coated reinforcement. Consideration must be given to the fact, that in designating galvanized reinforcement, all surrounding reinforcement and miscellaneous hardware, that is to be in touch with the galvanized reinforcement, must be galvanized, plastic or PVC coated.

17.10.2 Railroads

Railroad companies usually require steel sheet piling for excavations adjacent to railroad tracks. The Railroad Engineering Unit should be contacted for specific information regarding these requirements. This information should be obtained prior to the submission of Preliminary Bridge Plans.

Piers, that support bridges over railroads and that are located less than 25 ft. from the centerline of track, shall either be of solid shaft construction or shall be protected by a reinforced concrete crash wall that extends not less than 7 ft. above the top of rail. This will provide an allowance of 12 in. for future ballasting of the railroad tracks and for potential encroachment during construction or maintenance operations.

The crash wall shall be at least 3.5 ft. thick and shall connect with all the columns. The face of the crash wall shall extend a distance of at least 6 in. beyond the face of the columns on the side adjacent to the track and it shall be anchored to the columns and footings with adequate steel reinforcement.

NOTE: For more information, reference **Chapter 8, Part 2, Section 2.1.5 of the A.R.E.A. Manual For Railway Engineering.**

Footing designs within the theoretical railroad embankment line shall provide a 8.25 ft. minimum distance from any point on the rail to the side of the steel sheet piling used for support of tracks during construction.

17.10.3 Anchor Bolts

DDOT standards do not permit drilling holes for anchor bolts in rigid frame and T-type piers. The following steps shall be taken to insure proper construction clearances for anchor bolts.

Design drawings shall show (in a detail plan and a cross-section view) the relationships between the anchor bolts and the layers of reinforcement steel immediately under each bearing pad. Detail dimensions shall be given, locating the centers of the anchor bolts and reinforcement bars. Reinforcement bars adjacent to anchor bolts shall be so spaced as to allow the free installation of 3 in. diameter sleeves for setting anchor bolts.

The vertical rows and the horizontal layers of reinforcement steel shall be so spaced as to allow a minimum of 2 diameters clear space between bars to facilitate placing of the concrete.

17.11 Pier Selection

There are multiple criteria and considerations to be evaluated in selecting the most economical and structurally appropriate type of pier to be designed. These include:

- Separate Or Continuous Footings
- Footing Size
- Type Of Pier, Column, Solid Shaft Or Hammer-Head
- Number, Spacing And Size Of Columns
- Shaft Dimensions
- Cap Size

All of the forces that act on abutments also must be considered in the design of piers. In addition, stream, ice and drift forces must be considered.

17.12 Frame and Multi-Column Piers

Generally, one and two-column piers are not to be considered due to the lack of redundancy. In certain situations (i.e., very tall, very large columns), they may be viable. Minimum pier column dimensions are 30 in. and 36 in. preferred. Loading conditions may dictate a larger column size. Multiple-column piers are more economical in normal highway-over-highway construction. Depending on the pier length, three or more columns are usually used. Pier columns must be connected at the base of the columns with at least 36 in. high crash wall.

17.12.1 Reinforcement

Care should be used in spacing vertical column bars to avoid excessive interference with the pier cap reinforcement. Double rows of column bars or large-diameter columns should be considered to alleviate this problem. The spiral reinforcing shall be full height of column plus extend into the pier cap and the footing by a minimum of 18 in. and shall end with 1.5 turns at each end.

17.12.2 Construction Joints

If pier columns are over 30 ft. high, a construction joint should be placed at approximately mid-height.

17.12.3 Column Spacing

Columns should be spaced far enough apart to be appealing to the eye. The minimum center-to-center spacing is 15 ft. All piers columns shall be provided with crash walls at base of columns for underpass roadway structures and bridges over railroads.

17.12.4 Pier Caps

Pier caps should be proportionally sized to the columns. The minimum width of a cap is 33 in. or the width of the column, plus 4 in., whichever is greater.

17.12.5 Solid or Hollow Shaft Piers

In cases where space for large footings and multiple column piers is limited or columns are very high, solid or hollow shaft columns can be considered. Aesthetic treatment is preferred for massive concrete elements.

17.13 Pile Bents

Pile Bents are not recommended for use for permanent structures in the District; they may be considered for temporary structures. Pile Bents have most recently proven to be the most economical type of pier. This type of pier is generally most suited for structures crossing rivers, of low- to mid-level clearance and multi-span structures.

In cases where piles are subject to wet and dry cyclic exposure, only concrete piles with pile protection are used. The protective coating is applied to the surface of precast-prestressed concrete piles after the pile is cast. Steel shell piles are not used in water because of durability and environmental impacts involving maintenance cleaning and painting.

Generally, precast-prestressed concrete piles are more economical than fluted steel shells or pipe piles. Precast-prestressed concrete piles are fabricated in one piece to a length defined by the designer. Precast-prestressed piles are preferably not field spliced. Where piles can be barged to the construction site, piles in excess of 100 ft. in length can be used. In cases where piles must be driven to an elevation lower than the bottom of the cap to achieve bearing, cap heights may be increased to accomplish the design.

NOTE: The minimum pile size is 18 in. either in diameter or square.

17.14 Rock Riprap

The most common method of bank or slope protection is rock riprap. The sides of the bank or embankment are lined with large rocks to prevent erosion along the bank and at the toe. Appearance of the rock riprap is natural, and in time, vegetation will grow between the rocks. Construction must be accomplished in a prescribed manner to assure proper behavior. The factors to consider in the design of rock riprap protection are:

- Durability And Density Of The Rock
- Magnitude And Direction Of Stream Velocity
- Angle Of The Side Slopes
- Size Of The Rock
- Shape And Angularity Of The Rock

Filter blankets are used as reverse filters to prevent piping damage to the riprap caused by movement of small particles up through the larger stone as a result of decreased hydrostatic pressure from flowing water. Stone bank protection should terminate with a buried toe.

Design guides for estimating rock size for channel and stream bank protection are included in Chapter Three. The velocities noted in the Corps of Engineers Chart are considered to be the average velocity over the hydraulic section, and the velocity noted in the ASCE Chart is considered to be local velocity computed at a specific sub-area. The charts are considered simple approximations for estimating purposes only. Use the procedures in **FHWA publication HEC-11, Use of Riprap for Bank Protection**, for final design.

Specify a minimum 18 in. thick blanket for embankment protection and 24 in. thick for slope protection along stream banks and for streambeds. Where unusual problems are anticipated or the adequacy of ordinary practice is uncertain, a complete detailed design of the riprap gradation and filter blanket is recommended.

17.15 Integral Abutment Bridges

17.15.1 Characteristics of Integral Bridges

Integral abutment type bridge structures are simple or multiple span bridges that have their superstructure cast integrally with their substructure. Integral abutment bridges accommodate superstructure movements without conventional expansion joints.

With the superstructure rigidly connected to the substructure and with flexible substructure piling, the superstructure is permitted to expand and contract. Approach slabs, connected to the abutment and deck slab with reinforcement, move with the superstructure. At its junction to the approach pavement, the approach slab may be supported by a sleeper slab. If a sleeper slab is not utilized, the superstructure movement is accommodated using flexible pavement joints. Due to the elimination of the bridge deck expansion joints, construction and maintenance costs are reduced.

The integral abutment bridge concept is based on the theory that due to the flexibility of the piling, thermal stresses are transferred to the substructure by way of a rigid connection between the superstructure and substructure. The concrete abutment contains sufficient bulk to be considered a rigid mass. A positive connection with the ends of the beams or girders is provided by rigidly connecting the beams or girders and by encasing them in reinforced concrete. This provides for full transfer of temperature variation and live load rotational displacement to the abutment piling.

The connection between the abutments and the superstructure shall be assumed to be pinned for the superstructure's design and analysis. The superstructure design shall include a check for the adverse effects of fixity.

17.15.2 Criteria for Integral Abutment Bridge Design

The movement associated with integral abutment bridge design can be largely associated with thermal expansion and contraction of the superstructure. By definition, the length of an integral abutment structure shall be equal to the abutment centerline of bearing to abutment centerline of bearing dimension. This also applies to continuous span structure lengths with expansion bearings at the piers. This length of expansion mobilizes the horizontal passive earth pressure.

17.15.2.1 Expansion Provisions

- For bridge lengths 165 ft. or less, unless the highway pavement is rigid concrete, provision for expansion at the approach slab ends shall not be required.
- For bridge lengths over 165 ft. and up to 330 ft., provisions shall be made for expansion at the end of each approach slab by installation of a sleeper slab.
- For bridge lengths over 330 ft. and up to 460 ft., integral designs shall be approved by the Chief Transportation Engineer.
- For bridge lengths over 460 ft., integral abutments are not recommended at this time.

17.15.3 Design Procedure Guidelines

The following criteria shall be utilized in providing integral abutment bridge designs:

17.15.3.1 Hydraulics (Scour)

Integral abutment bridges provide fixity between the superstructure and substructure, and provide greater protection against translation and uplift than conventional bridges. The DDOT Bridge Scour Evaluation Program and Structure Inventory and Appraisal Inventory records shall be studied to verify scour potential at a project site. To address potential impact of a scour effect on proposed Integral abutment bridge sites, the following areas should be reviewed and analyzed where scour potential exists.

17.15.3.1.1 Stream Velocity

Any history of erosion or scour at the bridge site must be reviewed and a determination made if the new structure will alleviate any problems (alignment, restricted opening etc.) that may contribute to scour. Where a scour history is determined, the potential positive affects of an Integral abutment bridge should be noted. Scour information may be obtained by researching the DDOT Bridge Scour Evaluation Program and Structural Inventory and Appraisal coding records.

17.15.3.1.2 Bank Protection

Suitable slope protection construction, to provide protection against scour, must be provided. On all integral abutment bridges, geotextile bedding shall be used against the front face of the abutment, under the slope protection and down the slope a minimum of 6 ft.

17.15.3.2 Skew Angle

For all integral abutment bridge designs where skew angles are involved, the Designer shall utilize the 3-D FEA analysis to determine the actual skew angle. It is noted that the use of this structural tool does not preclude integral abutment bridge designs with skew angles greater than thirty degrees.

17.15.3.3 Foundation Types

- The abutment and pile design shall assume that the girders transfer all moments and vertical and horizontal forces that are produced by the superimposed dead load, live load plus impact, earth pressure, temperature, shrinkage, creep and seismic loads. The transfer of these forces shall be considered to be achieved after the rigid connection to the abutments is made. The rigid connection shall be detailed to resist all applied loads.

- All abutment substructure units shall be supported on a single row of piles. Cast-in-place (C.I.P.) or steel H piles may be used for structures with span lengths of 165 ft. or less. Only steel H piles should be used for structures with span lengths over 165 ft. When steel H piles are used, the web of the piles shall be perpendicular to the centerline of the beams regardless of the skew. This will facilitate the bending about the weak axis of the pile.
- To facilitate expansion, for bridge span lengths of 100 ft. or more, each pile at each substructure unit shall be inserted into a pre-bored hole that extends 8 ft. below the bottom of the footing. The cost of provision of pre-boring these holes, casings and cushion sand shall be included in the Unit Price Bid for the pile item. All details and notes required by the Foundation Design Report shall be placed on the plans. For bridge lengths under 100 ft, pre-boring is not required.
- The Designer must determine the practical point at which the embedded pile is determined to be fixed. The following steps may be followed to perform such an analysis.
  - Calculate the thermal movement demand. For a bridge structure with equal intermediate bent stiffness, the movement demand will be equal. The atmospheric temperature range, coefficient of expansion and the structure's length should be considered.
  - The plastic moment capacity of the embedded length of the pile (embedded in the concrete cap) must be calculated. As stated earlier, the pile shall be oriented for bending about the weak axis.
  - The column capacity must then be calculated.
  - The adequacy of the backwall to resist passive pressure due to expansion must be calculated.

- When C.I.P. piles are used, they must be pipe casings conforming to ASTM A252, Grade 2 with a minimum wall thickness of ¼ inch. This shall be noted on the plans.
- All piles shall be driven to provide proper penetration into soil strata where the required pile action is achieved, or to a minimum penetration of 20 ft. This is to avoid a stilt type effect, provide for scour protection and to provide sufficient lateral support to the pile.
- A pile bent configuration should be used for the integral abutment substructure detailing. For steel superstructure bridges, a minimum of one pile per girder shall be used.
- The piles shall be designed to be flexible under forces and moments acting on the abutment. They shall be designed for vertical and lateral loads and for bending induced by superstructure movement. The fixity between the superstructure and the pile top may be ignored.

- The initial choice of pile selection shall be based upon the recommendations that are contained in the Geotechnical Report. The axial loads shall be based upon the reactions from the superstructure design. This shall include the superstructure dead load, live load plus impact and the substructure dead load.
- Live load impact shall be included in the design of integral abutment piles. The total length for single span bridges and the end span length for multiple span length bridges should be considered.

17.15.3.4 Superstructure

- Adjacent prestressed box beams, prestressed concrete girders and structural steel beams may be used for integral abutment designs. They shall be analyzed to determine the stresses in the beams that will result from thermal movements. In prestressed box beams, such stresses shall be judged to be critical when the beams act by pulling an abutment with an approach slab. Mild reinforcement shall be added to the ends of prestressed box beams to resist such stresses.
- Steel superstructures may have their girders directly attached to the piles through the use of welded load plates. Other type connections, such as bolting the girder to the abutment, may also be used. Prestressed girders may be connected by doweling them to the abutments.
- Steel girders may be placed on plain elastomeric pads. The anchor bolts will pass through both the pad and the bottom flange of the girder. Another method is to use a longer bolt so that nuts may be placed above and below the bottom flange. The grade of the girder may be better controlled this way. Slotted holes should be used to allow better flexibility in aligning the girder.
- Slotted holes should also be used with the doweling of prestressed members to the abutments.

17.15.3.5 Abutments

- In integral abutment bridges, the ends of the superstructure girders are fixed to the integral abutments. Expansion joints are thus eliminated at these supports. When the expansion joints are eliminated, forces that are induced by resistance to thermal movements must be proportioned among all substructure units. This must be considered in the design of integral abutments.
- To facilitate the stress transfer from superstructure to substructure, abutments shall be placed parallel to each other and ideally be of equal height.

- The positive moment connection between the girder ends and the abutment provides for full transfer of temperature variation and live load rotational displacement to the abutment piling.
- To support the integral abutment, it is customary to use a single row of piles. The piles are driven vertically and none are battered. This arrangement of piles permits the abutment to move in a longitudinal direction under temperature effects.
- The most desirable type abutment is the stub type. It will provide greater flexibility and will offer the least resistance to cyclic thermal movements.

17.15.3.6 Piers

- Piers for integral bridges have similar design requirements and share common design procedures with the piers of a more traditional bridge. The primary distinguishing features of the piers for an integral abutment bridge involve their ability to accommodate potentially large superstructure movements and the sharing of longitudinal forces among the substructure units.
- As with integral abutments, the piers must also be designed to accommodate the movements of the superstructure. Thermal movements are usually the major concern, although superstructure movements, due to concrete creep and drying shrinkage, will also be present to some degree. Creep and shrinkage movements may be ignored for prestressed concrete girders; however, for longer bridges, these effects must also be considered in the design of the piers.
- As part of the overall structural system, integral abutment bridge piers will typically be required to carry a portion of the externally applied longitudinal and transverse loads on the bridge. In addition, thermal movements of the superstructure will induce forces as the piers attempt to restrain those movements.
- As the superstructure expands and contracts with seasonal temperature changes, and to a lesser extent, creep and shrinkage, the tops of the piers will be forced to undergo displacements relative to their bases. These displacements will produce curvatures in the columns that can be closely estimated based on the magnitude of the movements, the fixity conditions at the top and bottom of the columns and the height of the columns.
- Once curvatures are estimated, an effective column stiffness must be considered to compute internal moments and shears. A set of equivalent external forces, in equilibrium with the computed internal moments and shears, must be computed. This set of equivalent forces is used in subsequent analysis to represent the effects of superstructure movements on the piers.

- Forces induced by the distribution of the superstructure movements must be computed. Also, the distribution of externally applied loads to the substructure units must be estimated.
- As a general guideline to design an integral substructure, a spatial structural analysis with an integrated model of super and substructure should be performed.
- Similar to the design of a traditional pier, piers of integral abutment bridges are designed for load combinations. Often, load combinations involving temperature, creep and shrinkage control the design of integral abutment bridges, as opposed to combinations containing external loads only. A pier must be capable of undergoing the imposed superstructure movements while simultaneously resisting external forces.
- A bearing at a pier of an integral abutment bridge structure should only be fixed when the amount of expected expansion from the bearing to both abutments and adjoining pier is equal. All other cases should use expansion bearings.
- The following guidance shall be followed in determining the type of pier selection in integral abutment bridge designs:

17.15.3.6.1 Continuity at Piers

- The concrete deck slab must be physically continuous, with joints limited to sawcut control joints or construction joints. Distinction must be made between slab continuity and girder continuity at the piers.
- If, in accommodating the load transfer, girder continuity is deemed appropriate by the design, the superstructure shall be assumed continuous for live loads and superimposed dead loads only. Girders shall be erected as simple spans and made continuous by the addition of mild steel in the deck slab.
- Longer span integral bridges; i.e., those with spans over 100 ft. shall be detailed to provide a deck slab placement sequence if girder continuity is to be provided. Where applicable, casting of concrete diaphragms over the piers should be done concurrently with placement of the slab.
- When slab-only continuity is provided over the piers, girders are to be designed as simply supported for all loads.

17.15.3.6.2 Types of Piers

To design piers to accommodate potentially large superstructure movements, the following options are available:

- Flexible piers, rigidly connected to the superstructure

- Isolated rigid piers, connected to the superstructure by means of flexible bearings
- Semi-rigid piers, connected to the superstructure with dowels and neoprene bearing pads
- Hinged-base piers, connected to the superstructure with dowels and neoprene bearing pads.

17.15.3.6.3 Flexible Piers

- A single row of piles, with a concrete cap that is rigidly attached to the superstructure, provides a typical example of a flexible pier. This type of pier is assumed to provide vertical support only. The moments induced in the piles due to superstructure rotation or translation are small and may be ignored.
- A bridge constructed with flexible piers relies entirely on the integral abutments for lonitudinal stability and for resisting lonitudinal forces. Passive pressures behind the backwalls, friction, and passive pressures on the abutment piles should be mobilized to resist longitudinal forces.
- With this type of pier use, temporary lateral bracing may be required to provide stability during construction. Designers must consider a means to account for passive soil pressures in the vicinity of the backwalls.

17.15.3.6.4 Isolated Rigid Piers

- Rigid piers are defined as piers whose base is considered fixed against rotation and translation, either by large footings bearing on soil or rock, or by pile groups designed to resist moment. The connection to the superstructure is usually detailed in a way that allows free longitudinal movement of the superstructure, but restrains transverse movements. This type of detailing permits the superstructure to undergo thermal movements freely, yet allows the pier to participate in carrying transverse forces.
- With this class of pier, the superstructure is supported on relatively tall shimmed neoprene bearing pads. A shear block, isolated from the pier diaphragm with a compressible material such as cork, is cast on the top of the pier cap to guide the movement longitudinally, while restraining transverse movements.

- This type pier represents the traditional solution taken with steel girder bridges at so-called expansion piers. It offers the advantage of eliminating the stresses associated with superstructure thermal movements. It also provides piers that require no temporary shoring for stability during construction.
- In utilizing this system, additional consideration must be given to the detailing associated with the taller bearing pads and the detailing associated with the shear key. In addition, because the pier and the superstructure are isolated longitudinally, the designer must ensure that the bearing seats are wide enough to accommodate seismic movements.

17.15.3.6.5 Semi-Rigid Piers

- These piers are similar to rigid piers. Their bases are considered fixed by either large spread footings or pile groups; however, the connection of the piers to the superstructure differs significantly.
- In utilizing prestressed concrete girders that bear on elastomeric pads, a diaphragm is placed between the ends of the girders. Dowels, perhaps combined with a shear key between girders, connect the diaphragm to the pier cap. Compressible materials are frequently introduced along the edges of the diaphragm, and, along with the elastomeric bearing pads, allow the girders to rotate freely under live load.
- The dowels force the pier to move with the superstructure as it undergoes thermal expansion and contraction and, to a lesser extent, creep and shrinkage. Accommodation of these movements requires careful analysis during the design of the piers. Normally, the stiffness of the piers is assumed to be reduced due to cracking and creeping.
- There are several advantages to this type of pier: detailing is simplified, use of thin elastomeric pads are relatively inexpensive, temporary shoring is not required during construction, all piers participate in resisting seismic forces and the girders are positively attached to the piers. In addition, with many piers active in resisting longitudinal and transverse forces, the designer need not rely on passive soil pressures at the integral abutments to resist longitudinal forces.
- Design of semi-rigid piers is slightly more complicated because careful assessment of foundation conditions,

pier stiffness and estimated movement is required. In some situations semi-rigid piers are inappropriate. For example, short piers bearing on solid rock may not have adequate flexibility to accommodate movements without distress.

17.15.3.6.6 Hinged-Base Piers

- This type of pier may be used to avoid the need for an expansion pier in a situation where semi-rigid piers have inadequate flexibility. A "hinge" is cast into the top of the footing to permit flexibility of the column.
- Temporary construction shoring may be required, and additional detailing requirements at the top of the footing may increase cost; however, the designer should keep this alternate in mind under special circumstances where the other pier types are not feasible.

17.15.3.7 Wingwall Configuration

- In-Line wingwalls cantilevered off the abutments are the preferred arrangement for integral abutment construction. Wingwalls in excess of 13 ft. should be supported on their own foundation independent of the integral abutment system. In this case, a flexible joint must be provided between the wingwall stem and the abutment backwall.
- Flared walls cantilevered off of the abutments may be considered by the Designer on a case-by-case basis. The use of flared wingwalls should generally only be considered at stream crossings where the alignment and velocity of the stream would make in-line walls vulnerable to scour. Piles shall not be placed under any flared walls that are integral with the abutment stem.
- The U-walls shall preferably not measure more than 10 ft. from the rear face of the abutment stem. If U-walls greater than 10 ft. in length are required, the wingwall foundation should be separated from the abutment foundation. A flexible joint between the abutment backwall and wingwall stem should be provided. This type arrangement will maintain the abutment/pile flexibility so that the thermal movement of the superstructure is permitted.
- The distance between the approach slab and the rear face of the U-wall should preferably be a minimum of 4 ft. If the approach slab must extend to the U-wall, they shall be separated by a joint, filled with Resilient Joint Filler.

17.15.3.8 Horizontal Alignment

Only straight beams will be allowed. Provided that the beams are straight, structures on curved alignments will be permitted.

17.15.3.9 Grade

The maximum grade between abutments shall be 5 percent.

17.15.3.10 Stage Construction

Stage Construction is permitted. Special consideration shall be given to the superstructure's rigid connection to the substructure during concrete placement when staging construction. The superstructure should be secured, free from rotation, until all concrete, up to the deck slab, is placed.

17.15.3.11 Seismic Modeling

- The general concept behind modeling the seismic response of a bridge structure is to determine a force-displacement relationship for the total structure that is consistent with the ability of the structure to resist the predicted forces and displacements.
- Integral abutments shall be modeled to move under seismic loading in both the longitudinal and the transverse directions, thus distributing more transverse forces to the piers.

17.16 Construction Procedures

The following sequence is recommended when constructing integral bridges. This will reduce the effects of thermal movements on fresh concrete and control moments induced into the supporting pile system.

- Drive piling and pour the concrete to the required bridge seat elevation and install the rigid connection systems. Pour concrete for wingwalls concurrently.
- Set the beams/girders and anchor to the abutment. As an alternate, slotted bolt holes in the bottom flanges may be used. The slotted holes will aid the girder placement. Anchor nuts should not be fully tightened at this time. Free play for further dead load rotations should be accounted for.
- Pour the bridge deck in the sequence desired excluding the abutment backwall/diaphragm and the last portion of the bridge deck equal to the backwall/diaphragm width. In this manner, all dead load slab rotations will occur prior to lock-up, and no dead load moments will be transferred to the supporting piles.

- If utilizing anchor bolts, tighten anchor nuts and pour the backwall/diaphragm full height. Since no backfilling has occurred to this point, the abutment is free to move without overcoming passive pressures against the backwall/diaphragm. The wingwalls may also be poured concurrently.
- Place back of wall drain system and backfill in 6 in. lifts until the desired subgrade elevation is reached. Place bond breaker on abutment surfaces in contact with approach pavement.
- Pour the approach slab concrete starting at the end away from the abutment, progressing toward the backwall. If tension is the chief concern, the approach pavements should be poured in late afternoon so that the superstructure is contracting, and therefore not placing the slab in tension.
- A construction joint should be located at a distance of 6 inch from the back of the backwall between the approach slab and bridge slab. This will provide a controlled crack location rather than allowing a random crack pattern to develop. Corrosion coated dowels shall pass through the joint and shall be located near the bottom of the slab. This will keep the joint tight but still allow the approach slab to settle without causing tension cracking in the top of the slab.
- The excavation for the approach slabs shall be carefully made after compacted abutment embankment material is in place. The slabs shall be founded on undisturbed compacted material. No loose backfill will be allowed.
- To permit unhindered longitudinal movement of the approach slab, the surface of the sub-base course must be accurately controlled to follow and be parallel to the roadway grade and cross slope.
- A filter fabric or some type of bond breaker such as polyethylene sheets shall be placed on the finished sub-base course the full width of the roadway prior to placement of approach slab reinforcement.
- A lateral drainage system should be provided at the end of the approach slab adjacent to the sleeper slab.

NOTE: Suitable notes should be provided on the plans to incorporate these construction procedures.

17.17 Semi-Integral Abutment Design

A semi-integral abutment design structure is one whose superstructure is not rigidly connected to its substructure. It may be a single or multiple-span, continuous, structure whose integral characteristics include a jointless deck, integral end diaphragms, compressible backfill, and movable bearings. In this concept, the transfer of displacement due to the piles is minimized. The rotation is generally accomplished by use of a flexible bearing surface at a horizontal interface in the abutment. Horizontal displacements not eliminated in a semi-integral concept must still be considered in the design.

In lieu of conventional deck joint bridges, or where a full integral bridge is not desirable, semi-integral bridges may be considered. The foundations for this type structure shall be stable and fixed. A single row of piles should not be utilized. The foundation piles should be stiffened by inclusion of battered piles, or the foundation may be founded on bedrock.

The expansion and contraction movement of the superstructure should be accommodated at the roadway side of an approach slab. This type design shall only be used for symmetrical, straight beam structures. The geometry of the approach slab, design of the wingwalls and transition parapet, if any, must be compatible with the freedom required for the integral configuration (beams, deck, backwall and approach) to move longitudinally.